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CHAPTER 5. CHANNEL LOCATION, ALIGNMENT AND HYDRAULIC DESIGN

Location & Alignment

Channel alignment is an important feature of channel design. It should be selected with careful consideration given to all factors affecting its location, including an economic comparison of alternate alignments. The economic analysis should include all costs such as channel construction, rights-of-way, bridges, stabilizing measures and maintenance.

Many factors affect the planned alignment of a channel. Topography, the size of the proposed channel, the existing channel, tributary junctions, geologic conditions, channel stability, rights-of-way, existing bridges, required stabilization measures, farm boundaries, land use, and other physical features enter into this decision.

The shortest alignment between two points may provide the most efficient hydraulic layout but it might not meet all the objectives of the channel improvement or give due consideration to the limitations imposed by certain physical features. The shortest, well planned alignment should be used in flat topography if geologic conditions are favorable and if physical and property boundaries permit.

Alternate alignment should be considered in areas where geologic conditions present a stability problem. An alternate alignment may locate the channel in more stable soils. In some cases, the alignment of the existing channel may be satisfactory with only minor changes. An alignment resulting in a longer channel may, in a minor degree, help to alleviate stability problems. A longer channel will decrease the energy gradient which, in turn, will decrease the velocities and tractive forces. A meander channel will increase Manning's coefficient "n" which will reduce velocities. A meander channel, however, presents the problem of erosion at the curves in the channel which, in erosive soils, may require structural protection such as jetties, riprap, brush maps, etc. Suggested minimum radii of curvature are cited in NEH, Section 16[2/], Chapter 6, for channels of indicated size and gradient. Guidance for alignment consideration in design of higher velocity channels is presented later in this chapter under Supercritical Flow. When alternate alignments and designs are not feasible, or do not assure a stable channel, stabilization structures should be included in the design. Such structures are discussed in a later section of this chapter.

Where feasible, the alignment should be planned to make use of existing, adequate bridges and road structures which have many years of remaining life. From a technical standpoint, bridges and road structures in poor condition should not influence the alignment. Care should be exercised to minimize cases of isolating parts of fields from the rest of the farm, but good alignment should not be sacrificed to follow all farm boundaries. Long reaches of channel should be located in the low areas, particularly where drainage is a problem. Long tangents should be used wherever possible. In meander channels, good alignment should not be sacrificed to use the maximum amount of the old channel.

The above discussion on alignment deals with subcritical flow only. Supercritical flow requires consideration of some of the most complex problems in hydraulics and may require model studies for a basis of design criteria. A discussion on supercritical flow may be found in NEH, Section 5[20].

Hydraulic Design

Criteria and procedures for hydraulic design of all types of channels are presented for subcritical and supercritical flow conditions.

Subcritical Flow

Design procedures for subcritical flow conditions for a limited range of functional and hydraulic conditions commonly encountered in agricultural drainage work are described in NEH, Section 16[2], Chapter 6. The following procedures apply to all conditions of subcritical flow:

Step 1. - - Determine the design discharge for all channel reaches. Use methods given in Chapter 4. Where overbank flow contributes a significant amount of water, it is best to assume that the discharge changes abruptly at arbitrary points within the channel reach and that the discharge remains constant between these points. As a guide to selection of these arbitrary points, Q should not be decreased more than 10 percent at any such point. As an example, consider a 10,000-foot reach of channel with overbank flow and no significant tributaries. Q = 3000 cfs at the downstream end and 2000 cfs at the upper end. Here it would be logical to reduce Q by 200 cfs increments at each of four evenly spaced points within the reach. (The last 200 cfs reduction would be at the upstream end of the reach.)

Step 2. - - Determine the water surface elevation at the downstream end of proposed construction. (Assuming that the approximate channel alignment has already been set.) The method for determining this water surface elevation varies with the outlet condition:

1. Water stages in the outlet are independent of channel discharge. For this condition it is necessary to have stage data on the outlet stream or tidal outlet. Two water surface profiles must usually be run on the tributary channel--one with the water surface at the outlet at the highest possible elevation within the design flood frequency to insure capacity, and one with the water surface at the lowest level within the above limitation to insure channel stability with the resulting increased velocity.

2. Outlet is at a control point where critical flow exists. In this case, the control point establishes the water surface elevation at critical depth.

3. Outlet is at a point in the stream where the water surface can be established. In this case, where the downstream channel is prismatic, the water surface elevations at the outlet to the improved reach may be established by the methods outlined in T. R. 15, or by an assumption of uniform flow at that point if the downstream prismatic channel is sufficiently long and is not affected by grade changes.

   In the event that the downstream channel is non-prismatic, it will be necessary to begin water surface profile calculations at a point considerably downstream using the methods outlined in T. R. 14 or NEH, Section 5[20/], Supplement A, to determine the water surface elevation at the starting point for construction.

Step 3. - - Establish the water surface control line by the methods outlined in Chapter 6, NEH, Section 16[2/]. Such a control line is helpful in establishing a desirable hydraulic grade line and subsequently, the design invert grade. In setting the hydraulic control line, consideration must be given to freeboard requirements and, where applicable, to additional depth required for superelevation of the water surface and flow in the unstable range.

Freeboard is defined as the additional channel depth required for safety above the calculated maximum depth of water. Freeboard is exclusive of additional depth computed for superelevation or turbulence in the unstable range. Freeboard for trapezoidal channels at sub-critical flow should be equal to or greater than 20% of the depth at design discharge but not less than one foot. (10% is satisfactory for rectangular lined channels at sub-critical flow.)

In closely controlled irrigation canals or channels where overbank flooding is permissible at frequent intervals, the above freeboard criteria may be disregarded.

Where possible, it is advisable to keep the design water surface below the level of natural ground. There is usually no objection to containing the freeboard in fill. Occasionally, such a great saving can be accomplished by containing part of the design discharge in fill over low areas in the profile that it is economical to do so even at the expense of close construction control on the dike.

Curve radii should be sufficiently great to limit superelevation of the water surface to one foot above computed depth of flow or 10% of water surface width, whichever is the least.

The amount of superelevation may be determined as follows for subcritical flow in trapezoidal channels:

$$s = \frac{V^2 \ (b + 2\ zd)}{2\ (gR - 2zV^2)}$$

where the terms are as defined in the glossary.

Channels whose energy gradients at design flow have a slope at or near critical ($0.7s_c < s_o < 1.3s_c$) will require additional depth as follows:

$$H_w = 0.25\ d_c \left[1 - 11.1 \left(\frac{s_o}{s_c} - 1 \right)^2 \right]$$

where the terms are as defined in the glossary.

The above criterion applies to flows with velocities slightly greater than critical as well as to sub-critical flow in the unstable range. Where possible, grades and/or cross sections should be adjusted to avoid the unstable range.

Step 4. - - Select values of Manning's coefficient "n". The estimation of realistic values of the roughness coefficient "n" is an important factor in channel design. The value "n" indicates the net effect of all factors, except grade and hydraulic radius, causing retardation of flow in the reach of channel under consideration. The estimation of "n" warrants critical study and judgement in the evaluation of the factors affecting its value. The primary factors are: irregularity of the surfaces of the channel sides and bottom, variations in shape and size of cross sections, obstructions, vegetation, and alignment of the channel.

A systematic procedure for the estimation of "n" values is contained in NEH, Section 5[20/], Supplement B. This procedure should be used and supplemented by any other applicable data.

Table 1 in SCS-TP-61 [21/] lists a range of "n" values for various channel linings and conditions, and contains a method of estimating "n" values for various vegetal linings.

The design capacity of a channel should be based on the "n" value anticipated after the channel has aged, giving consideration to the degree of maintenance that can reasonably be expected. When stability is in question, the stability of the channel should be checked with the "n" value anticipated immediately after construction.

Step 5. - - Determine the allowable side slopes by procedures given in Chapter 6 of this guide or NEH, Section 16, Chapter 6, if applicable.

Step 6. - - Determine allowable velocities or tractive forces for the various reaches, depending on which procedure is to be used to check channel stability against flowing water. These values will need to be considered when selecting channel sizes and slopes in step 7. Use procedures given in Chapter 6 of this guide. Since the depth of flow is required for use in both the velocity and tractive force procedures, this step will have to be done concurrently with step 7.

Step 7. - - Determine the size and shape of channel needed. Since the side slopes have been determined by other considerations, the design problem becomes the determination of the required depth and bottom width.

Trial cross sections may be selected by assuming uniform flow conditions and solving for the depth or width by using Manning's equation. Several combinations of depth and width should be evaluated.

Generally, the most economical channel will be one in which the hydraulic grade line approaches the water surface control line determined in step 2. When this is planned, the slope of the water surface control line may be used as the value of s in Manning's formula. The depth of flow determined for the reach will give the position of the channel bottom. The slope of the channel bottom may be made parallel to the water surface control line. If care is used in selecting the proportions of the cross section, only a slight change in depth of flow or in bottom width at the ends of each reach will result. For the above condition, the effect of transitions on the water surface profile will be small and for all practical purposes the water surface control line becomes the true water surface profile for the channel. (See Fig. 6-13, NEH, Section 16[2/])

Step 8. - - Compute the water surface profile for the best apparent cross section. Hydraulic design methods outlined in NEH, Section 16[2/], Chapter 6 are based on the assumption that Manning's formula defines the slope of the hydraulic gradient. This assumption is valid for channels where changes in velocity head from one reach to the next are insignificant, and will provide a short cut to water surface profile calculations.

The generally accepted method of computing the water surface profile is outlined in T. R. 15. Water surface profiles must begin at the downstream end of the work and proceed upstream. Where reaches are long, the upstream part of the reach will usually approach a condition of uniform flow.

Often, the completed water surface profile will point up the need for reproportioning the cross section to provide more capacity or more economy. Usually an adequate and economic cross section may be arrived at within two trial solutions.

Curves in alignment. - - Often it is necessary to re-evaluate curves in earth channels after the hydraulic design is otherwise complete. With velocities known it is possible to determine the minimum curve radius permissible without protection, as outlined under "Channel Stability." The decision between complying with this minimum radius and shortening the radius and providing rock riprap bank protection on the curve then becomes a matter of economics. Cost of right-of-way, severance or structure removal may be greater than the cost of protecting a tight curve. In very flat topography ($s_o < 0.001$ ft./ft.) Table 6.1, NEH Section 16[2/], may be used to determine minimum radius of unprotected curves.

Side drainage. - - Major tributaries on which work is to be done as a part of the project are usually brought in at channel invert grade. Water surface profiles above the junction on both tributaries may be calculated as stated above.

Minor tributaries are usually brought in at an elevation above the channel invert. The channel bank, thus exposed, must be protected by rock riprap, concrete, pipe over-pour or other methods.

Overland flow entering the channel should be concentrated where possible and brought in at selected locations. Where it is not practical to concentrate such flow, the channel bank should be protected or vegetated, unless the soil materials are sufficiently resistant to the erosive forces applied to remain stable.

Where low areas must be drained into the channel, it may be necessary to install conduits through a dike and attach automatic drainage gates to the conduits to prevent outflow from the channel.

Channel entrance. - - Flood channels are usually terminated at the upstream end where:

1. The discharge is sufficiently small that the existing facility has adequate capacity, or

2. The benefits from additional length of channel improvement will not justify the cost thereof.

In the former case, the work may usually be terminated by a simple transition from the constructed channel to the existing channel. Occasionally it will be necessary to install a drop spillway structure to reconcile the grade difference.

In the latter situation, it may be necessary to construct wing dikes to collect the upstream flow and concentrate it into the constructed channel.

Earth channels with grade control structures. - - This type of protection is particularly satisfactory where existing channels have plenty of capacity.

Energy gradient is controlled by proportioning the weir notch so that the head necessary to operate the weir at design discharge corresponds to the uniform flow depth upstream from the structure, thus defining the water surface profile. In this case, energy dissipation is accomplished at the structures by changing part of the horizontal velocity to vertical and dissipating the vertical velocity head in the plunge pool. Drops that are exceptionally low in relation to tailwater (particularly submerged drops) are likely to be inefficient and their basins must be quite long to accomplish energy dissipation.

Three types of drop spillway structures are commonly used in Service work:

1. Type B Drop Spillway

2. Type C Drop Spillway

3. Box Inlet Drop Spillway

Criteria for the design of Type B and Type C drops may be found in NEH Section 11[4]. Type B drops have shorter basins, are not so deeply buried, and, consequently, are cheaper where hydraulic conditions permit their use. Type B structures are not designed for submergence or high tailwater to fall relationships.

The Type C structure is more tolerant of high tailwater and submergence, but must be longer and more deeply buried below the downstream channel invert.

Criteria for the design of the Box Inlet drop spillway may be found in SCS-TP-106[22]. This structure is useful when a long weir crest is needed in a relatively narrow channel, when deep excavation would be difficult and expensive, when a sizable pool in the basin would constitute a health hazard, and on high drops where potential sliding constitutes a stability problem.

Lined channels. - - Rock, concrete or other trapezoidal lining is usually provided for channels at subcritical flow where velocities are sufficiently great that the bottom and banks of an unprotected channel would be unstable. When velocities are supercritical, rock lining will usually be uneconomical because of the large size rock and thick section required for stability.

Hydraulic design procedure is similar to that for unprotected earth channels. Additional depth may be required on curves because of superelevation of the water surface.

Because of the relatively high cost of rock and filter blanket or other lining material, it is best to design for maximum hydraulic radius within limits of available depth, bank stability and reasonable excavation equipment width.

Earth channels with bank protection only. - - This type of protection is used when unprotected banks would be unstable, but the bottom is naturally erosion resistant.

Hydraulic design procedure is the same as for unprotected earth or rock-lined channels except for the composite friction coefficient.

Supercritical Flow

Hydraulic design procedure for lined channels at supercritical flow may differ quite radically from that for subcritical channels. Here, the principal concern is with capacity and, within limits, the greater the velocity the more economical the cross section, as all supercritical channels not excavated in rock will require lining.

Although the procedure for the selection of trial cross sections, alignment, and grade is similar to that for subcritical channels, final design differs in the following ways:

1. Stability against erosion is no longer a factor, having been accomplished by mechanical protection.

2. Side inlets for small discharges can usually be brought in at any level above channel grade without special protection.

3. Junctions for channels with supercritical flow must be carefully designed. Supplemental model studies may be needed if the proposed design differs radically from junctions on which model study information is already available.

4. Superelevation on curves is an important factor in design.

5. Trapezoidal sections should be avoided on curves.

6. For supercritical flow, water surface profiles must be run in a downstream direction.

Design Procedure. - - Trial cross sections and grades may usually be selected on the basis of uniform flow characteristics. In supercritical reaches it is unlikely that changes will be needed when the water surface profile is computed. Curves will nearly always require additional depth. Superelevation of the water surface may be determined using the methods described below:

1. For rectangular channels at subcritical velocity, or at supercritical velocity where a stable transverse slope has been attained by use of an upstream easement curve (spiral easement or compound curve).

   $$S = \frac{3V^2b}{4gR} \qquad \text{(See glossary)}$$

2. Supercritical velocity - simple curve.

   $$s = 1.2 \frac{V^2b}{gR}$$

   Location of the first point of maximum depth on the outside wall may be determined by the following formula:

   $$\theta = \cos^{-1}\left[\frac{R - \frac{b}{2}}{R + \frac{b}{2}} \cos\beta\right] - \beta$$

   where the terms are as defined in the glossary.

Either spiral easements or compound curves may be employed to reduce superelevation in accordance with the following criteria.

3. Compound Curve Criteria

The complete curve is to consist of three sections; a central section with radius R_c and an approach and terminal section each with a radius R_t equal to twice R_c. This produces a superelevation in the zone of the first maximum equal to one-half the normal superelevation produced in a simple curve whose radius is equal to that of the central section.

$$R_t = 2\ R_c$$

The length of each transition curve in terms of the central angle

$$\theta_t = \tan^{-1} \frac{b}{R_t \tan \beta}$$

Compound curves shall be used under the following conditions:

When necessary to limit superelevation to one foot allowable maximum.

When two successive curves occur with an intervening tangent less than 1000' in length.

4. Spiral Easement Curve

Such easement curves other than the constant radius transition may be used provided that a simple disturbance pattern is produced, and the maximum wave height on the outside wall at the beginning of the curvature of the main curve is equal to:

$$S = \frac{V^2 b}{2gR}$$

Trapezoidal lined channels are not recommended for curved alignment at supercritical flow because of the difficulty in predicting wave run-up on the sloped banks.

Freeboard. - - Minimum freeboard of 0.2 times the depth should be provided for rectangular channels at supercritical flow and 0.25 times the depth for trapezoidal supercritical channels.

Junction structures. - - Design of channel junction structures has not been covered in any standard hydraulic reference. Specific studies have resulted in criteria and guidance on analysis and design of elements of the problem. Model study may be necessary to confirm or refine design of important structures.

Model study of confluences has made evident the need to make the junction with the two flows as nearly parallel as possible. This reduces velocity and momentum components (which cause waves normal to the direction of combined flow) to a minimum.

Having accomplished this, application of the momentum principle to the junction will provide a reliable analysis for design. Further guidance for junction design is contained in Fig. 5-1, and "Hydraulic Model Studies for Whiting Field Naval Air Station," [23] prepared by the Soil Conservation Service.

Channel entrance. - - Some type of structure with weir control is needed at the entrance to the lined section. Such a structure may be either the straight weir or folded weir type. Folded weirs require a drop to prevent submergence.

Transitions. - - Simple transitions in bottom width of rectangular channels may usually be handled by limiting the angle between the transition walls to ten degrees or less. Transitions between rectangular and trapezoidal sections, particularly where the trapezoidal section is in earth, are more complex. The attached paper on transitions (see Appendix I) has been used in the design of several transitions that function satisfactorily.

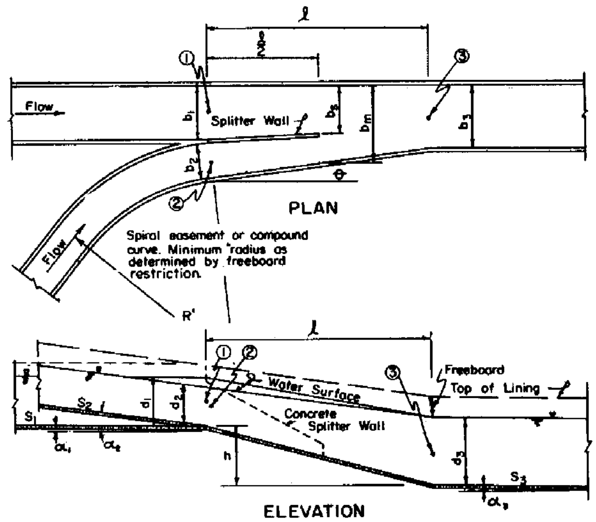
Channel outlet. - - The SAF Basin is the most satisfactory outlet structure where rectangular R/C channels at supercritical flow discharge into an earth section. It is also permissible to use a R/C transition to a trapezoidal rock-lined section. The rock lining should extend a sufficient distance downstream at zero or very low gradient that velocities are reduced to those permissible in the earth materials in the bed and banks of the channel.

Bridges and Culverts. - - Crossings over relatively narrow, rectangular R/C channels are easily accomplished with R/C single span box culverts. These, when bottom slab and sidewalls match those of the channel, have no hydraulic effect. This is also true for clear span bridges over rectangular or trapezoidal channels. Losses caused by bridge piers or interior walls of multiple cell box culverts may be determined by the momentum method. (See Appendix II)

NOMENCLATURE

A	=	cross-sectional area of water prism	$ft.^2$
b	=	base width of channel section	ft.
c	=	constant - see Chart A	--
d	=	vertical depth of water	ft.
g	=	acceleration due to gravity (32.2)	$fps.^2$
h	=	difference in invert elevation between any two sections	ft.
ℓ	=	length of channel reach	ft.
n	=	coefficient of roughness in Manning's formula	--
P_a	=	average wetted perimeter between two sections	ft.
P_m	=	wetted perimeter at middle section	ft.
Q	=	rate of discharge	cfs.
R_a	=	average hydraulic radius between two sections	ft.
R_m	=	hydraulic radius at middle section	ft.
R'	=	radius of curve to centerline	ft.
S_c	=	critical slope of channel invert	--
S	=	channel invert slope	--
V_a	=	average velocity between two sections	fps.
V_m	=	velocity at middle section	fps.
Z	=	side slope of trapezoidal section (horizontal to vertical)	--
θ	=	angle of merging channels	deg.
α	=	invert slope angle	deg.
F	=	Froude number	--

Numerical subscripts refer to appropriate channel members.



1. TRAPEZOIDAL CHANNELS

Constant Width Main Channel

$$\frac{Q_3^2}{gA_3}\cos\alpha_3+\left(\frac{b_3}{2}+\frac{Zd_3}{3}\right)\left(d_3^2\right)\cos^2\alpha_3=\frac{Q_1^2}{gA_1}\cos\alpha_1+\frac{Q_2^2}{gA_2}\cos\alpha_2\cos\theta+\left(\frac{b_1}{2}+\frac{Zd_1}{3}\right)\left(d_1^2\right)\cos^2\alpha_1+A_a h-\frac{P_a \ell n^2 V_a^2}{2.21\,R_a^{1/3}}$$

Unequal Width Main Channel

$$\frac{Q_3^2}{gA_3}\cos\alpha_3+\left(\frac{b_3}{2}+\frac{Zd_3}{3}\right)\left(d_3^2\right)\cos^2\alpha_3=\frac{Q_1^2}{gA_1}\cos\alpha_1+\frac{Q_2^2}{gA_2}\cos\alpha_2\cos\theta+\left(\frac{b_3}{2}+\frac{Zd_1}{3}\right)\left(d_1^2\right)\cos^2\alpha_1+(b_3+Zd_m)\,d_m h-\frac{P_m \ell n^2 V_m^2}{2.21\,R_m^{1/3}}$$

Note: For flows at supercritical velocities these equations may be used only for preliminary design. Final design must be based on results of supplemental hydraulic model studies.

2. RECTANGULAR CHANNELS

Constant Width Main Channel

$$\frac{Q_3^2}{gA_3}\cos\alpha_3+\frac{b_3 d_3^2}{2}\cos^2\alpha_3-\frac{b_3 d_3 h}{2}=\frac{Q_1^2}{gA_1}\cos\alpha_1+\frac{Q_2^2}{gA_2}\cos\alpha_2\cos\theta+\frac{b_3 d_1 h}{2}+\frac{b_3 d_1^2}{2}\cos^2\alpha_1-\frac{P_a \ell n^2 V_a^2}{2.21\,R_a^{1/3}}$$

Unequal Width Main Channel

$$\frac{Q_3^2}{gA_3}\cos\alpha_3+\frac{b_3 d_3^2}{2}\cos^2\alpha_3-\frac{b_3 d_3 h}{2}=\frac{Q_1^2}{gA_1}\cos\alpha_1+\frac{Q_2^2}{gA_2}\cos\alpha_2\cos\theta+\frac{b_3 d_1 h}{2}+\frac{b_1 d_1^2}{2}\cos^2\alpha_1+\frac{(b_3-b_1)}{2}\left(d_2^2\right)\cos^2\alpha_2-\frac{P_m \ell n^2 V_m^2}{2.21\,R_m^{1/3}}$$

SPECIAL CASE

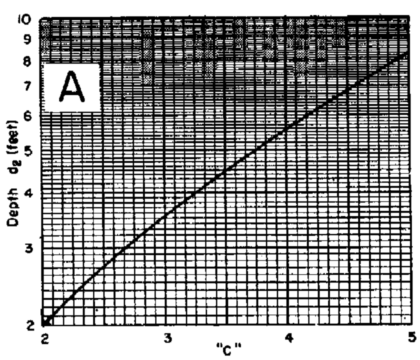
Limiting Criteria - Rectangular Channels
Supercritical Velocity Flow in All Branches.

Constant Width Main Channel

$$\frac{Q_3^2}{gA_3}+\frac{b_3 d_3^2}{2}-\frac{b_3 d_3 h}{2}=\frac{Q_1^2}{gA_1}+\frac{Q_2^2}{gA_2}+\frac{b_3 d_1 h}{2}+\frac{b_3 d_1^2}{2}-\frac{P_a \ell n^2 V_a^2}{2.21\,R_a^{1/3}}$$

Unequal Width Main Channel

$$\frac{Q_3^2}{gA_3}+\frac{b_3 d_3^2}{2}-\frac{b_3 d_3 h}{2}=\frac{Q_1^2}{gA_1}+\frac{Q_2^2}{gA_2}+\frac{b_3 d_1 h}{2}+\frac{b_1 d_1^2}{2}+\frac{(b_3-b_1)}{2}\left(d_2^2\right)-\frac{P_m \ell n^2 V_m^2}{2.21\,R_m^{1/3}}$$



Criteria

$\tan\alpha < 0.1$

Convergence angle θ less than 10°

$F=\frac{V}{\sqrt{gd}} \geq 1.2$

$\theta \leq 10^\circ$

$1.2 \geq \frac{d_1}{d_2} \geq 0.8$

Height of sidewall at (3) $= 1.2\,d_3 + 0.25\,d_{c_3}\left[1-11.1\left(\frac{S_3}{S_{c_3}}-1\right)^2\right]+1.2\,\frac{V_2^2 b_2}{gR_2}$

Height of splitter wall
upstream = $d_{1,2}$ max.
downstream = 6" min.

$\ell = cV_2\left[b_1+b_2-b_3\right]^{1/3}$

$b_3 \geq 0.8\,(b_1+b_2)$

$b_s=\frac{b_1\,(b_1+b_2+b_3)}{2(b_1+b_2)}$ when $b_1 > b_2$

$b_e=\frac{b_2\,(b_1+b_2+b_3)}{2(b_1+b_2)}$ when $b_1 < b_2$

FIGURE 5-1
HYDRAULICS JUNCTION STRUCTURES

REFERENCE: FLOW ANALYSIS AT OPEN CHANNEL JUNCTIONS; U.S.C.E.-L.A. 1947

APPENDIX I TO CHAPTER 5

ON

CHANNEL PLANNING AND DESIGN

TRANSITIONS

Important Transitions Where it is Necessary to Conserve Head

Circumstances requiring a change in channel section occur frequently. The crossing of roads, the changing of grades, and many topographic conditions provide situations where acceleration or deceleration of flow is necessary to meet required changes of cross section.

Adequate design of transition structures to provide for gradual changes of the flow section is important because at such places the capacity of the whole system is frequently determined. Poor transition design not only vitiates good channel construction but may cause undesirable backwater effects.

Although transition design is based on the Bernoulli and Continuity equations, experience plays a very significant part. Model studies and observations of many actual structures have indicated several rules to be followed. They are:

1. The water surface should be smoothly transitioned to meet end conditions.

2. The water surface edges should not at any section converge at an angle greater than 28° with the center line, nor diverge at an angle greater than 25°.

3. In well designed transitions, losses in addition to friction should not exceed .10 h_v for convergence and .20 h_v for divergence.

4. In general it is desirable to have bottom grades and side slopes meet end conditions tangentially. (Usually this rule must be violated when critical depths are approached going from an earth channel to a lined ditch.)

To outline the specific steps in designing a transition, it is desirable to reduce Bernoulli's equation to a more convenient form. Equation (5.1-1) reads

$$\frac{V_1^2}{2g} + \frac{P_1}{W} + z_1 = \frac{V_2^2}{2g} + \frac{P_2}{W} + z_2 + h_f \qquad \text{(Eq. 5.1-1)}$$

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If the flow takes place on sufficiently flat slopes so that P equals WY, then the terms $\frac{P}{W} + Z$ are exactly equal to the elevation of the water surface for all elements. If a fall of surface downstream is taken as positive, Bernoulli's equation becomes:

$$\Delta W.S. = \Delta h_v + h_f + \text{impact} \qquad \text{(Eq. 5.1-2)}$$

For simplicity, the term "impact" is used as a measure of losses due to change in direction of stream lines in both converging and diverging transitions. In a converging transition, these losses are truly due to impact as stream lines impinge against the converging walls. In a diverging transition, losses are caused primarily by eddy currents resulting from negative pressures along the diverging walls.

In relatively short transition structures the ordinary friction losses given by the Manning formula are small compared with the impact losses and the head loss in the transition is very nearly $.10 \Delta h_v$ for inlets and $.20 \Delta h_v$ for outlets. Equation (5.1-2) reduces finally to $\Delta W.S.$ equals $1.10 \Delta h_v$ for inlets and $\Delta W.S.$ equals $.80 \Delta h_v$ for outlets. Friction loss must be considered for long transitions and for velocities in excess of 20 fps.

These simple relationships assisted by the continuity equation

$$Q = A_1 V_1 = A_2 V_2 \qquad \text{(Eq. 5.1-3)}$$

form the basis of all transition design. Detailed steps to be followed in the computation follow.

1. From the quantity of flow compute the velocities and velocity heads at the end sections.

2. Compute the overall change in water surface from

   $\Delta W.S.$ equals $1.10 \Delta h_v$ (inlets)

   $\Delta W.S.$ equals $.80 \Delta h_v$ (outlets)

   (neglecting ordinary friction loss)

3. Construct a smooth curve to represent the water surface having the computed change in elevation and tangent to the surfaces at the ends. Two reverse parabolic curves are good. However, any smooth curve can be used.

4. Mark this surface curve at 6 to 10 stations (depending on the size of the structure), and tabulate the total ΔW.S. from the beginning of the transition to each station. The distance between these stations is assumed at first and then adjusted until the desired conditions are reaches.

5. Compute Δh_v from $\frac{\Delta \text{W.S.}}{1.1.0}$ or $\frac{\Delta \text{W.S.}}{0.80}$ for each station and

   evaluate each h_v and V.

6. From Q equals AV, obtain the cross sectional area required at each section.

7. Assuming a bottom grade line to meet end conditions, list depths at each station.

8. Evaluating the average width from $\frac{A}{d}$, select side slopes to make the water surface converge smoothly according to the requirements of rules 2 and 4. If this cannot be done by adjustment of side slopes along, the transition may be lengthened, the bottom grade line changed or the water surface may be varied. A juggling of these controls will finally produce the required results.

Care should be taken in designing the transition when the velocity goes through critical. On inlet transitions the bottom should be raised gradually until the critical depth is reached and just beyond it should drop as fast as possible. In this way the critical depth is very unstable and unless this is done the critical depth may not come at the computed location and cause improper loading in the channel below.

It is possible to design an outlet transition from a subcritical to a super critical depth without going through a hydraulic jump, but it is better to avoid this condition if possible.

9. To allow for the friction loss, compute P and R for each section and compute the rate of friction loss, f, for flow at each section from

$$f = \frac{n^2 \quad v^2}{2.208R^{4/3}} \qquad \text{(Eq. 5.1-4)}$$

10. Then the friction loss between any two sections equals the average value of f for the two sections times the length between sections.

11. Then the water surface and bottom at each section must be dropped an amount equal to the summation of the values found in step 10. (From the beginning to the point in question.)

These steps are illustrated in the following examples.

Design an inlet transition from an earth channel, with a discharge of 314 C.F.S., bottom width 18.0 feet, depth 4.30 feet, side slopes 2:1, area 114.40 square feet and a velocity of 2.75 feet per second, to a rectangular concrete lined channel, with a width of 12.5 feet, depth 4.220 feet, area 52.70 square feet and a velocity of 5.97 feet per second.*

Critical depth is not involved in this transition.

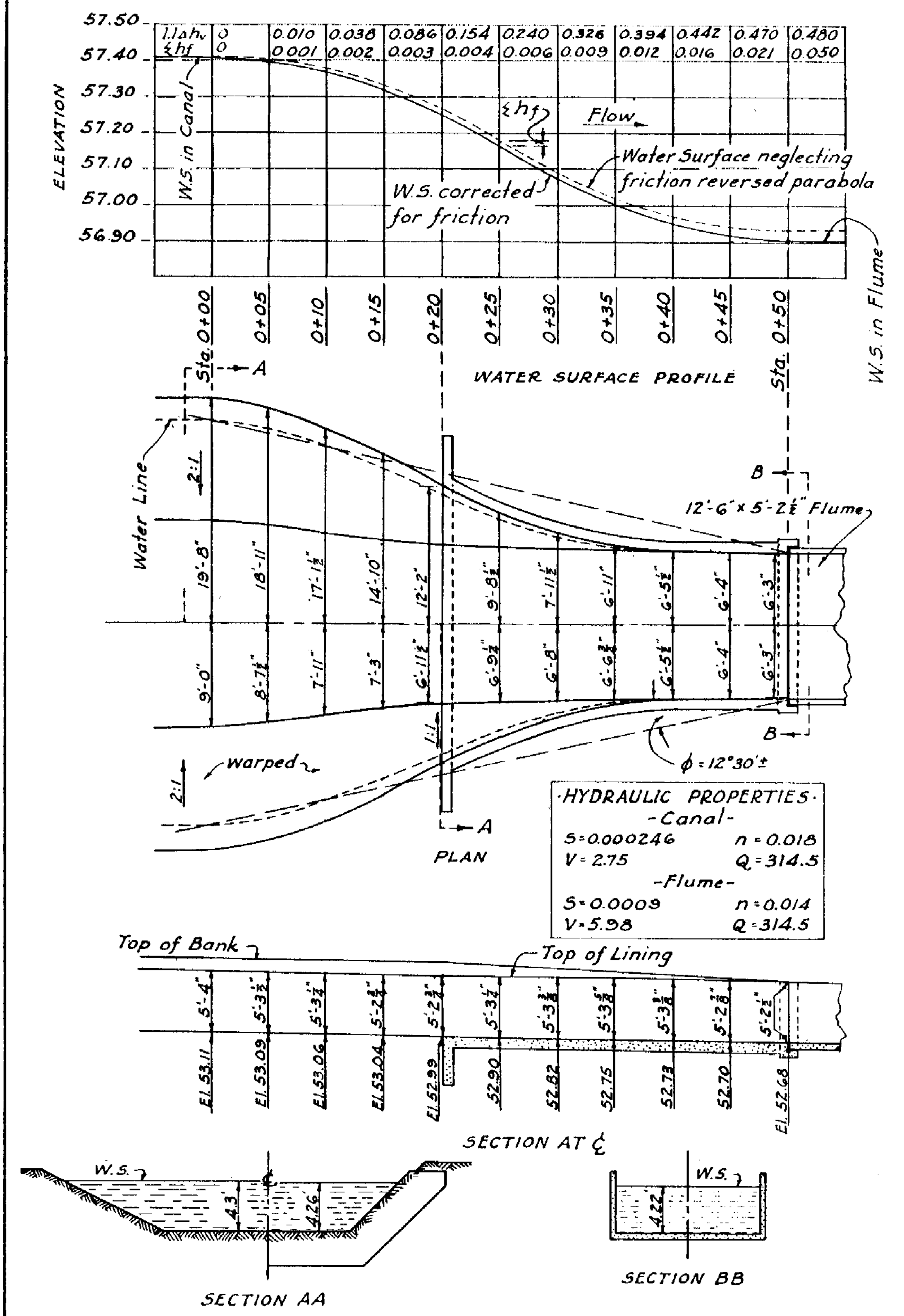
Line	Item	**0 + 00	0 + 05	0 + 10	0 + 15	0 + 20	0 + 25	0 + 30	0 + 35	0 + 40	0 + 45	0 + 50
1	Δ WS = Drop in WS		0.010	0.038	0.086	0.154	0.240	0.326	0.394	0.442	0.470	0.480
2	Δh_v = WS $\div$ 1.1		0.009	0.035	0.079	0.140	0.218	0.296	0.357	0.401	0.427	0.436
3	$h_v = 0.117 + \Delta h_v$		0.126	0.152	0.196	0.257	0.335	0.413	0.474	0.518	0.544	0.553
4	V	2.75	2.85	3.13	3.55	4.07	4.64	5.15	5.52	5.77	5.91	5.97
5	Area = Q $\div$ V	114.40	110.50	100.60	88.75	77.40	67.88	61.20	57.10	54.60	53.30	52.70
6	0.5T=Half width at WS	17.600	17.000	15.427	13.460	11.228	9.139	7.717	6.847	6.458	6.315	6.25
7	0.5B=Half bottom width	9.000	8.625	7.917	7.250	6.958	6.771	6.667	6.563	6.458	6.315	6.25
8	0.5T+0.5B=Average width	26.600	25.625	23.344	20.710	18.186	15.910	14.384	13.410	12.916	12.630	12.500
9	d = Area $\div$ Ave. width	4.30	4.309	4.310	4.280	4.260	4.264	4.252	4.253	4.225	4.220	4.220
10	f = Friction slope	0.00015	0.00017	0.00020	0.00026	0.00034	0.00046	0.00061	0.00076	0.00083	0.00087	0.00090
11	h_f = 5f (Use ave. f)		0.00080	0.00090	0.00115	0.00150	0.00200	0.00270	0.00345	0.00400	0.00425	0.00445
12	Σh_f		0.00080	0.00170	0.00285	0.00435	0.00635	0.00905	0.01250	0.01650	0.02075	0.02520
13	WS Elev=57.41-Δ WS-Σh_f	57.410	57.399	57.370	57.321	57.252	57.164	57.075	57.003	56.951	56.919	56.905
14	Grade = WS Elev - d	53.110	53.090	53.060	53.041	52.992	52.900	52.823	52.750	52.726	52.699	52.685
15	0.5 T - 0.5 B	8.600	8.375	7.510	6.210	4.270	2.368	1.050	0.284			
16	Side slopes	2.000	1.945	1.744	1.447	1.000	0.554	0.247	0.067			
17	Height of lining = H	5.330	5.295	5.270	5.234	5.228	5.265	5.287	5.305	5.274	5.236	5.205
18	0.5W-0.5B=Side slope x H	10.660	10.310	9.210	7.575	5.228	2.920	1.305	0.354			
19	0.5 W = 0.5 top width	19.660	18.935	17.127	14.825	12.186	9.691	7.972	6.917	6.458	6.315	6.250
20	0.5 W to nearest 1/2 in.	19'8"	18'11"	17'1½"	14'10"	12'2"	9'8½"	7'11½"	6'11"	6'5½"	6'4"	6' 3"

* This transition was taken from "Design of Important Transitions, not Involving Critical Flow or the Hydraulic Jump" by Julian Hinds, from "Transactions of the American Society of C.E." Vol. 92, Page 1430 et seq.

** In this case the stations are given in distances direct.

FIGURE 5.1-1

TYPICAL TRANSITION TO RECTANGULAR FLUME



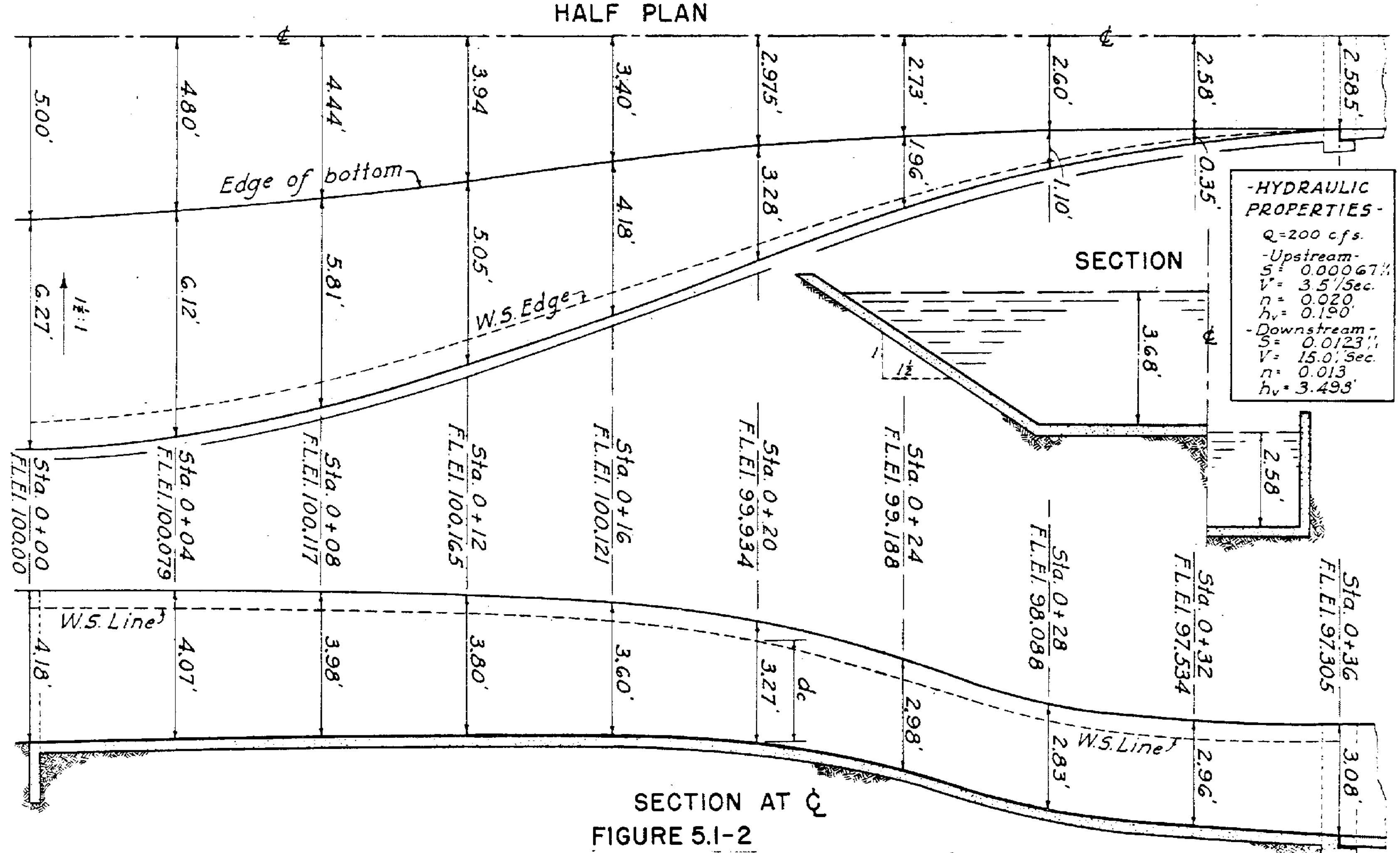
Design an inlet transition from an earth channel with a discharge of 200 C.F.S., bottom width 10.0 feet, depth 3.68 feet, side slopes 1-1/2:1, area 57.11 square feet and a velocity of 3.5 feet per second to a rectangular concrete lined channel with a width of 5.17 feet, depth 2.58 feet, area 13.34 square feet and a velocity of 15.0 feet per second.

In this example the depth is greater than the critical in the earth channel and less than critical in the concrete lined channel. In such a transition the bottom should have a slope less than the critical up to the point of critical depth, at this point it should break sharply to a slope greater than critical. In this example the critical depth is at sta. 6 or 20 feet from the earth channel.

Line	Item	Station									
		*1	2	3	4	5	6	7	8	9	10
1	Δ W.S. = 1.1 h_v	0.00	0.030	0.080	0.21	0.45	0.96	1.98	3.20	3.58	3.64
2	$\Delta h_v = \Delta$ W.S. $\div$ 1.1	0.00	0.027	0.073	0.191	0.409	0.873	1.800	2.910	3.255	3.310
3	$h_v = 0.190 + \Delta h_v$	0.190	0.217	0.263	0.381	0.600	1.063	1.999	3.100	3.445	3.500
4	$V = \sqrt{2 g h_v}$	3.50	3.74	4.11	4.95	6.21	8.27	11.31	14.12	14.89	15.00
5	A = Q/V	57.14	53.48	48.66	40.40	32.21	24.18	17.68	14.16	13.43	13.33
6	0.5 T (Measured)	10.52	10.18	9.52	8.32	7.00	5.755	4.40	3.48	2.87	2.585
7	0.5 b (Measured)	5.00	4.80	4.44	3.94	3.40	2.975	2.73	2.60	2.58	2.585
8	Average Width = (.5 T + .5b)	15.52	14.98	13.96	12.26	10.40	8.73	7.13	6.08	5.45	5.17
9	d = A/Av. Width	3.68	3.57	3.48	3.30	3.10	2.77	2.48	2.33	2.46	2.58
10	**f=Friction Slope=$n^2 v^2 \div 2208 R^{4/3}$	.00028	.00034	.00043	.00068	.00119	.00249	.00541	.00985	.01206	.0123
11	h_f = 4 (Average f)		.00124	.00154	.00220	.00372	.00736	.01580	.03052	.04384	.04872
12	Σ hf		.00124	.00278	.00498	.00870	.01606	.03186	.06238	.10622	.15494
13	W.S. Elev.=103.68-Δ W.S.-Σh_f	103.68	103.649	103.597	103.465	103.221	102.704	101.668	100.418	99.994	99.885
14	Grade = W.S. Elev. - d	100.00	100.079	100.117	100.165	100.121	99.934	99.188	98.088	97.534	97.305
15	Side Slopes	1.5:1	1.51:1	1.46:1	1.33:1	1.16:1	1.01:1	0.66:1	0.38:1	0.12:1	Vertical
16	Height of Lining H	4.18	4.07	3.98	3.80	3.60	3.27	2.98	2.83	2.96	3.08
17	.5W - .5b = side slopes x H	6.27	6.12	5.81	5.05	4.18	3.28	1.96	1.10	0.35	0.0
18	.5W = .5 top width of lining	11.27	10.92	10.25	8.99	7.58	6.26	4.69	3.70	2.93	2.585

* By plotting it was found a distance of 4 ft. between stations fit the desired conditions.

** n equals 0.013



SECTION AT ℄

FIGURE 5.1-2

TYPICAL INLET TRANSITION

Design an outlet transition from a concrete lined channel with a discharge of 200 C.F.S., bottom width 2.00 feet, depth 3.00 feet, side slopes 1 to 1, area 15.00 square feet and a velocity of 13.33 feet per second to another concrete lined channel, bottom width 6.00 feet, depth 2.25 feet, side slopes 1 to 1, area 18.57 square feet and a velocity of 10.77 feet per second, both depths being subcritical.

Due to the high velocities involved, the transition should not diverge greater than about 1 to 10 for each side. If in proportioning the side and bottom it is necessary to change the surface curve, it should be borne in mind that the velocity head changes rapidly with a small change in velocity for high velocities.

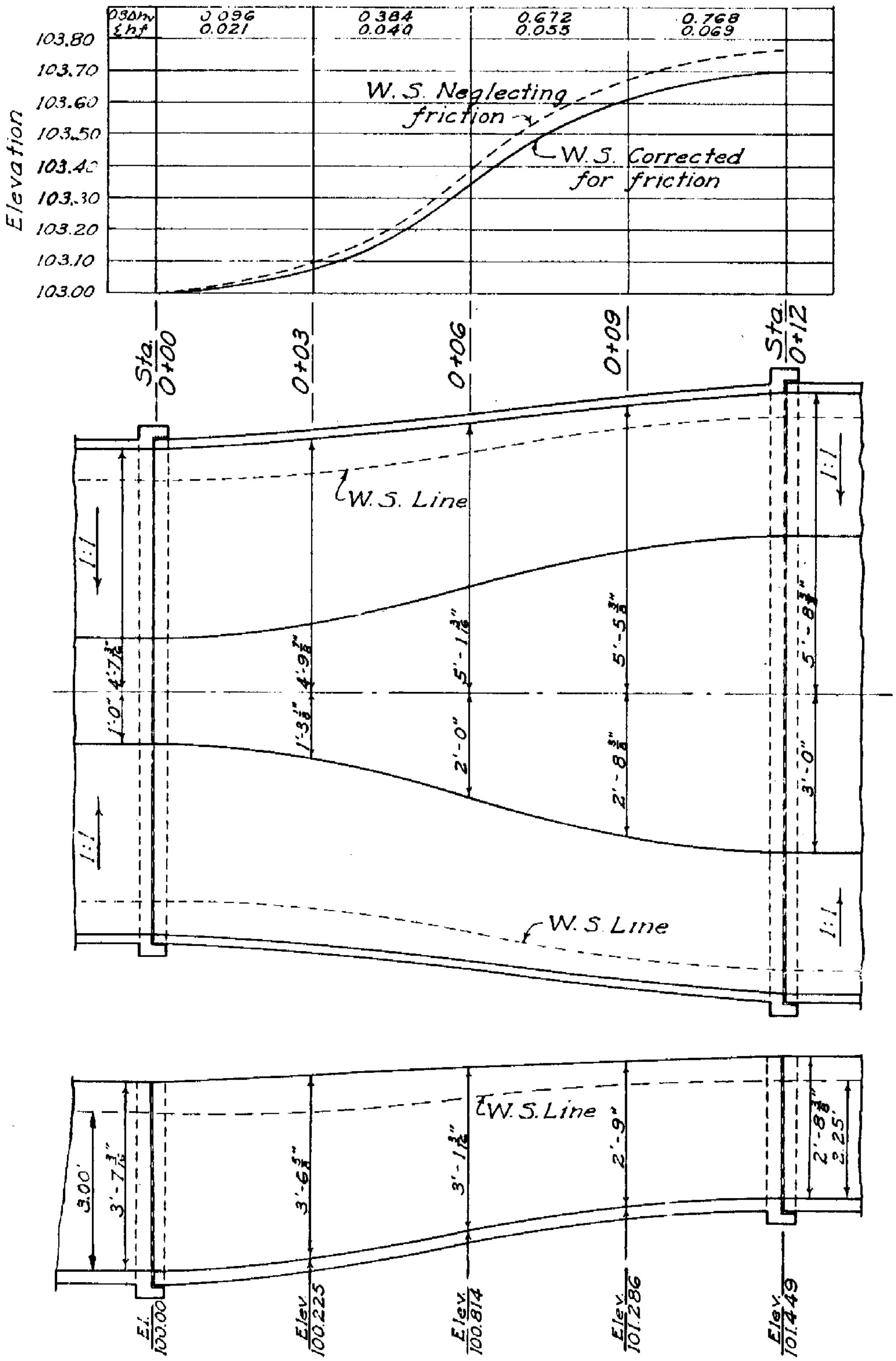
Line	Item	Station				
		*1	2	3	4	5
1	Δ W.S. = Rise in W.S.		0.096	0.384	0.672	0.768
2	$\Delta h_v = \Delta$ W.S. $\div$ 0.80		0.120	0.480	0.840	0.960
3	$h_v = 2.76 - \Delta h_v$	2.760	2.640	2.280	1.920	1.800
4	$V = \sqrt{2gh}$	13.33	13.09	12.11	11.11	10.77
5	Area = $Q \div V$	15.00	15.34	16.52	18.00	18.57
6	d scaled	3.00	2.85	2.53	2.33	2.25
7	Average width = $A \div d$	5.00	5.38	6.53	7.73	8.25
8	.5 T = Half top width	4.00	4.11	4.53	5.03	5.25
9	.5 b = Half bottom width	1.00	1.26	2.00	2.70	3.00
10	**f = Friction slope	.0072	.0068	.0057	.0047	.0044
11	h_f = 3 (Average f)		.0210	.0188	.0156	.0137
12	Σh_f =		.021	.040	.055	.069
13	W.S. El. = 103 + Δ W.S. $- \Sigma h_f$	103.00	103.075	103.344	103.616	103.699
14	Grade = W.S. El. - d	100.00	100.225	100.814	101.286	101.449
15	Side Slopes	1.0	1.0	1.0	1.0	1.0
16	Height of lining = H	3.60	3.55	3.10	2.75	2.70
17	.5W - .5B = side slopes x H	3.60	3.55	3.10	2.75	2.70
18	.5W = .5 top width of lining	4.60	4.82	5.10	5.45	5.70

* By plotting it was found that a distance of 3 ft. between stations fit the desired conditions.

** n equals 0.012

FIGURE 5.1-3

TYPICAL OUTLET TRANSITION



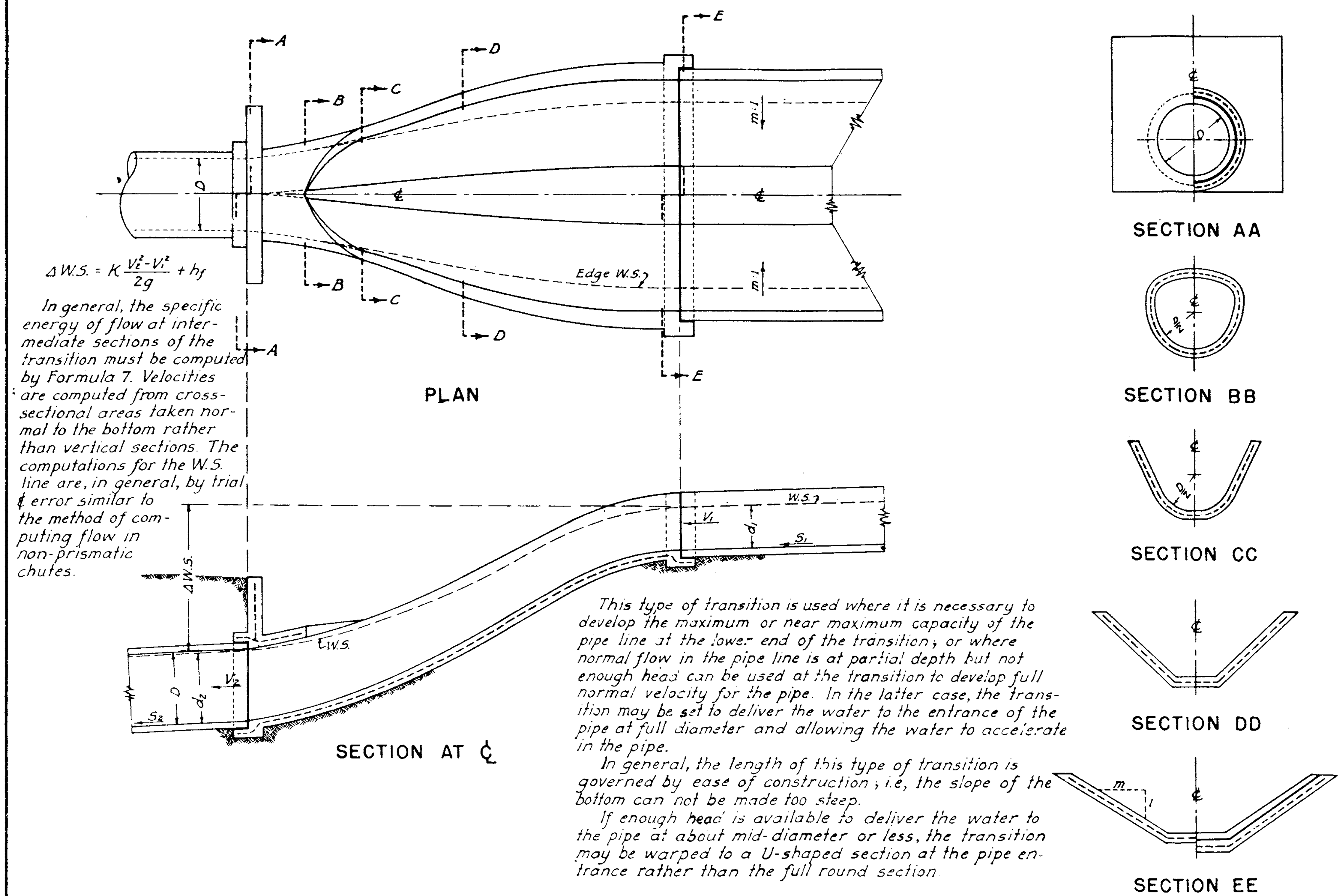
HYDRAULIC PROPERTIES

Q = 200 c.f.s.

Upstream		Downstream	
s = 0.0072 %	n = 0.012	s = 0.0044 %	n = 0.012
V = 13.33 '/sec.	h_v = 2.760'	V = 10.77 '/sec.	h_v = 1.800'

FIGURE 5.1-4

TYPICAL INLET TRANSITION TO PIPE LINE



In general, the specific energy of flow at intermediate sections of the transition must be computed by Formula 7. Velocities are computed from cross-sectional areas taken normal to the bottom rather than vertical sections. The computations for the W.S. line are, in general, by trial & error similar to the method of computing flow in non-prismatic chutes.

This type of transition is used where it is necessary to develop the maximum or near maximum capacity of the pipe line at the lower end of the transition; or where normal flow in the pipe line is at partial depth but not enough head can be used at the transition to develop full normal velocity for the pipe. In the latter case, the transition may be set to deliver the water to the entrance of the pipe at full diameter and allowing the water to accelerate in the pipe.

In general, the length of this type of transition is governed by ease of construction; i.e, the slope of the bottom can not be made too steep.

If enough head is available to deliver the water to the pipe at about mid-diameter or less, the transition may be warped to a U-shaped section at the pipe entrance rather than the full round section.

Less Important Transitions

This type of design is used when head is not at a premium.

The elevation of the water surface at each end is known. No attempt is made to trace out the water surface curve at intermediate points. The sides are straight lines and can be made vertical when going from an earth channel to a rectangular or circular section and vice versa. If the side slopes of the two sections are different, they should be gradually warped to meet the end conditions. The bottom should be laid in tangent to the grade at each end.

In the absence of more specific knowledge the length of the transition should be such that a straight line joining the flow line at the two ends of the transition will make an angle of about 12 1/2° with the axis of the structure.

Neglecting friction the losses can be taken as $0.15 \Delta h_v$ for inlet, and $0.25 \Delta h_v$ for outlet transitions.

In transitioning from an earth channel to a lined channel with a velocity greater than the critical, the earth channel should be contracted at the entrance to the transition sufficient to develop critical depth, and not develop scouring velocities above. The bottom of the transition should drop rapidly from the entrance and connect tangent to the grade on the channel below.

The procedure in designing such a transition is:

1. Compute the length.

2. Compute the change in water surface from:

   W.S. equals $1.15 \Delta h_v$ (inlets)

   W.S. equals $0.75 \Delta h_v$ (outlets)

   (neglecting ordinary friction loss)

To illustrate this procedure let it be required to design a transition from an earth channel carrying 100 second feet, bottom width 12.6 feet, depth of water 2.1 feet, total depth 2.6 feet, side slopes 1-1/2 to 1 and an average velocity of 3.0 feet per second to a concrete lined channel with a bottom width of 3.0 feet, depth of water 1.72 feet, total depth 2.0 feet, side slopes 1-1/2 to 1 and an average velocity of 10.43 feet per second.

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The normal depth in the concrete channel of 1.72 feet is less than the critical, therefore it is necessary to develop a velocity greater than the critical. The earth channel should be contracted to develop critical depth, without an excess drawdown effect in the earth channel above. In the earth channel under consideration it is necessary to contract the bottom to a width of 8.5 feet, the side slopes being 1-1/2 to 1.

d_c equals 1.44 feet

V_c equals 6.5 feet per second

1. Length of transition

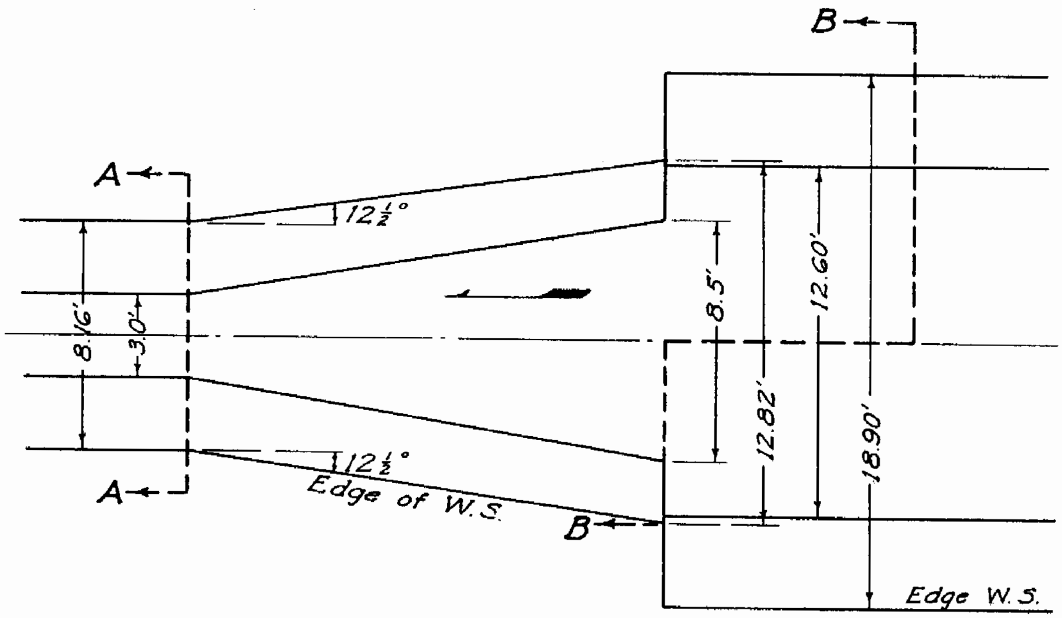
   2.33 x cot 12-1/2^{o} equals 2.33 x 4.51 equals 10.5 feet say 10.0 ft.

2. W.S. equals 1.15 (Δh_v)

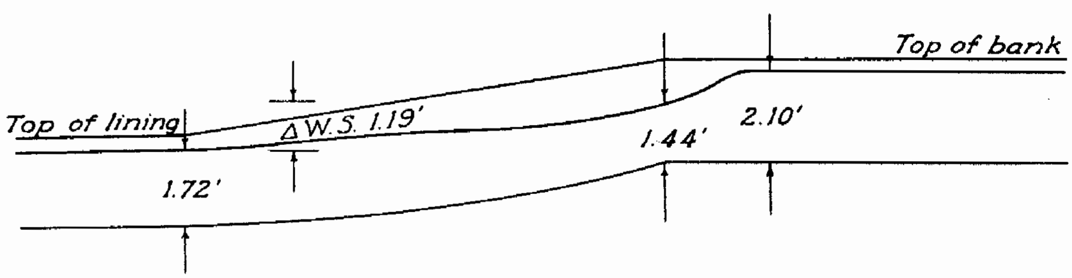
   equals 1.15 (1.035) equals 1.19 feet (See Fig. 5.1-5)

FIGURE 5.1-5

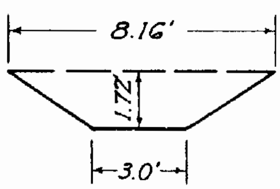
TYPICAL TRANSITION WITH STRAIGHT SIDES



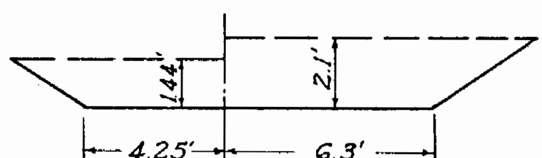
Plan Showing Bottom & Lines at W. S.



Side Elevation Showing Bottom & Approx. W. S.



Section A-A



Section B-B

APPENDIX II TO CHAPTER 5

MOMENTUM METHOD OF DETERMINING BRIDGE PIER LOSS *

Flow past an obstruction has been divided into three types which follow roughly "Class A and B" flow as defined by Yarnell, and "Class C" flow as indicated by Yarnell and defined herein. The definitions as given by Koch and Carstanjen for the three flow conditions follow:

"Class A" flow is defined as a flow condition whereby critical flow within the constricted bridge section is insufficient to produce the momentum required downstream. It is apparent that for this type of flow, the bridge section is not a "control point" and, therefore, the upstream water depth is controlled by the downstream water depth plus the total losses incurred in passing the bridge section.

"Class B" flow is defined as a flow condition whereby critical flow within the constricted bridge section produces or exceeds the momentum required downstream. When this condition exists, the upstream water depth is independent of the downstream water depth, being controlled directly by the critical momentum required within the constricted bridge section and the entrance losses.

"Class C" A special form of "Class B" flow occurs when the upstream water is flowing at a subcritical depth and containing sufficient momentum to overcome the entrance losses and produce a supercritical velocity within the constricted bridge section.

The drawing on the following page, entitled "Bridge Pier Losses by the Momentum Method" shows the water surface profiles and momentum curves for the three classes of flow.

Momentum, as referred to above, is defined as total momentum or the total of static and kinetic momentum, and may be written as $m + \frac{Q^2}{gA}$

where m = total static pressure of the water at a given section in pounds

Q = discharge in cubic feet per second

g = acceleration of gravity in feet per second per second

A = channel cross-sectional area in square feet.

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* Data derived from "Report of Engineering Aspects, Flood of March 1938, Los Angeles, California," - Appendix I, Theoretical and Observed Bridge Pier Losses - U. S. Engineer's Office, Los Angeles, California, - May 1949 and from "Approximate Method Determines Bridge Pier Loss," by G. M. Allen, Jr., in March 1953, Civil Engineering.

The unit weight of water (w) should appear in each term, but since it would cancel in the final equations, it has been assumed equal to unity, dimensions being pounds per cubic foot.

Based on experiments under all conditions of open channel flow where the channel was constricted by short flat surfaces perpendicular to flow, such as bridge pier, Koch and Carstanjen found that the total kinetic loss was equal to $\frac{A_0 Q^2}{A_1 g A_1}$ where A_0 is the area of the obstruction on the upstream surface and A_1 is the water area in the upstream unobstructed channel. For circular nose piers, Koch and Carstanjen show that 2/3 of $\frac{(A_0 Q^2)}{(A_1 g A_1)}$ should be used. It is apparent that the static pressure m_0 against the upstream obstructed area is not effective downstream, whereas the static pressure against the downstream obstructed area is effective downstream. Therefore, if we let the subscripts 1, 2, and 3 represent conditions upstream, within and downstream of the constricted section, respectively, we may write the general momentum relationship as follows:

Total upstream momentum minus the momentum loss at entrance must equal the total momentum within the constricted section, or

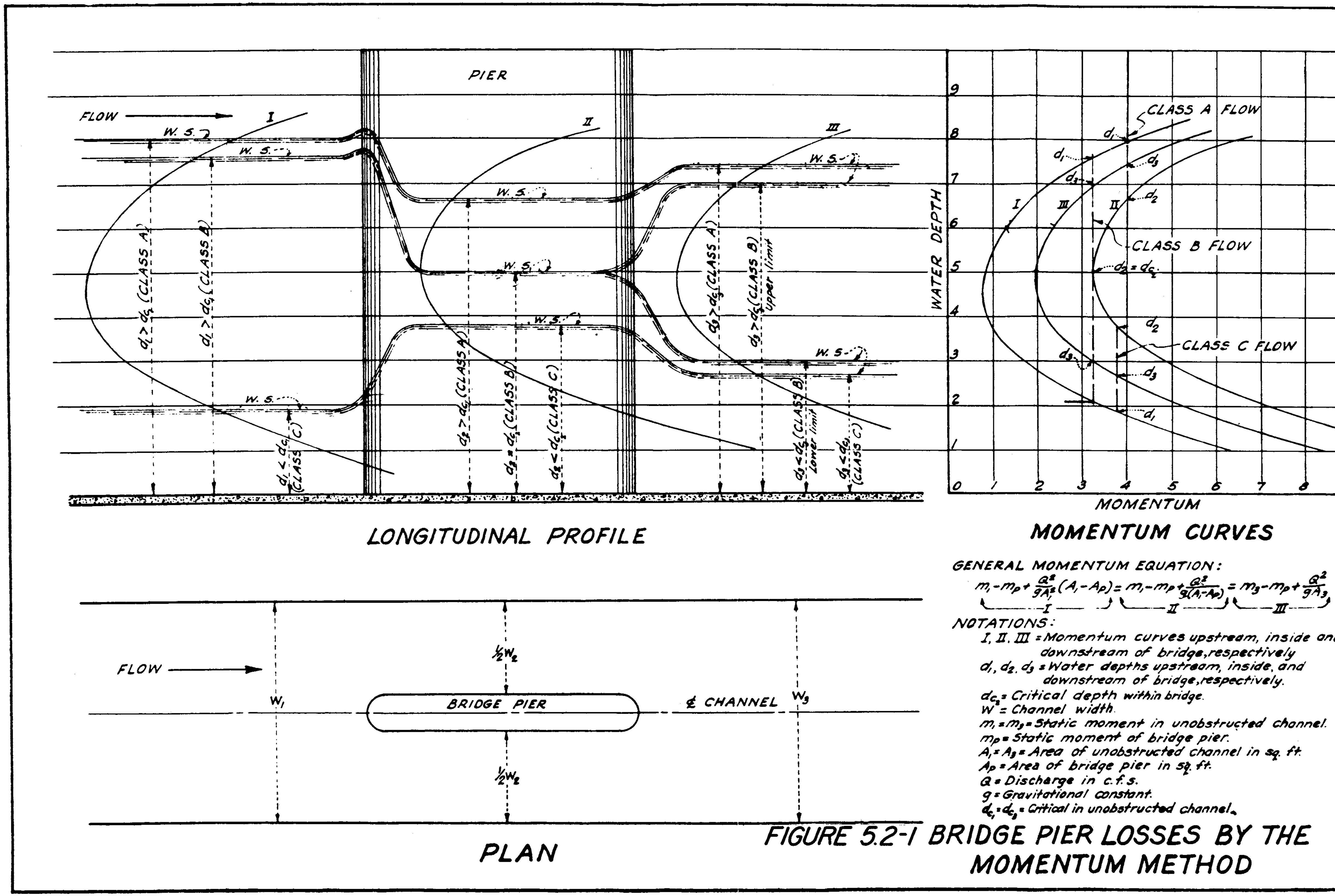
$$m_1 - m_0 + \frac{Q^2}{gA_1} - \frac{A_0}{A_1}\frac{Q^2}{gA_1} = m_2 + \frac{Q^2}{gA_2}, \text{ or}$$

$$m_1 - m_0 + \frac{Q^2}{gA_1^{\,2}}(A_1 - A_0) = m_2 + \frac{Q^2}{gA_2}$$

Total momentum within the constricted section plus static pressure on the downstream obstructed area must equal the total momentum in the downstream channel, or

$$m_2 + \frac{Q^2}{gA_2} + m_0 = m_3 + \frac{Q^2}{gA_3}, \text{ or}$$

$$m_2 + \frac{Q^2}{gA_2} = m_3 - m_0 + \frac{Q^2}{gA_3}$$



GENERAL MOMENTUM EQUATION:

$$\underbrace{m_1 - m_p + \frac{Q^2}{gA_1^2}(A_1 - A_p)}_{I} = \underbrace{m_1 - m_p + \frac{Q^2}{g(A_1 - A_p)}}_{II} = \underbrace{m_3 - m_p + \frac{Q^2}{gA_3}}_{III}$$

NOTATIONS:

I, II, III = Momentum curves upstream, inside and downstream of bridge, respectively

d_1, d_2, d_3 = Water depths upstream, inside, and downstream of bridge, respectively.

d_{c_2} = Critical depth within bridge.

W = Channel width.

$m_1 = m_3$ = Static moment in unobstructed channel.

m_p = Static moment of bridge pier.

$A_1 = A_3$ = Area of unobstructed channel in sq. ft.

A_p = Area of bridge pier in sq. ft.

Q = Discharge in c.f.s.

g = Gravitational constant.

$d_{c_1} = d_{c_3}$ = Critical in unobstructed channel.

FIGURE 5.2-1 BRIDGE PIER LOSSES BY THE MOMENTUM METHOD

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The general momentum equation follows:

$$m_1 - m_0 + \frac{Q^2}{gA_1^{\ 2}}(A_1 - A_0) = m_2 + \frac{Q^2}{gA_2} = m_3 - m_0 + \frac{Q^2}{gA_3}$$

The above equations cannot be solved as presented, and it is necessary that a simpler method be used. The total sum of momentum and hydrostatic pressure for each section (Fig. 5.2-3) for equal depths of flow past each section should first be determined. Using equal depths, $A_1 - A_0 = A_2 = A_3 - A_0$ (or $A_1 = A_3$) and $M_1 - M_0 = M_2 = M_3 - M_0$ (or $M_1 = M_3$).

Also, for equal depths, the sum of momentum and hydrostatic pressure for each section I, II, and III is:

$$I = M_1 - M_0 + \frac{Q^2 (A_1 - A_0)}{gA^2{}_1}$$

$$II = M_1 - M_0 + \frac{Q^2}{g\ (A_1 - A_0)}$$

$$III = M_1 - M_0 + \frac{Q^2}{gA_3} = M_1 - M_0 + \frac{Q^2}{gA_1}$$

where, for equal depths, $A_1 = A_3$

The values for equations I, II, and III are determined for various depths, both subcritical and supercritical. A curve for each section is plotted using the depth as the ordinate and the values from columns I, II, and III as the abscissa (see Fig. 5.2-4). A vertical line passed through the three curves gives a graphic solution of the equations, as it gives, for equal momentum, the corresponding depths of flow.

This vertical line must intersect a minimum of five depth values, and preferably six. Drwg. No. 7-N-Eng. 248, page 3, shows the depth of flow and its indicated class. If only one value is intersected on curve II, the flow is critical at Section II.

Values of "d" on the lower portions of Curves I and III are used for supercritical flow, and on the upper portions for subcritical flow.

Backwater computations will determine either a flow depth at Section I or III, depending upon type of flow conditions, and the curves give a direct solution, as the vertical line must pass through the known depth on Curve I or III, and must also pass through Curve II. If this vertical line does not pass through Curve II, it is possible that the momentum of the given depth is not great enough for the flow to pass the obstruction, and a change in the computed depth must be made. The flow would then be critical at Section II, as the critical depth is the depth at which the momentum and pressure is the minimum.

<u>Example</u>

Given a trapizoidal channel, base-width 16 feet, side-slopes 1-3/4:1 and capacity Q = 5000 c.f.s.

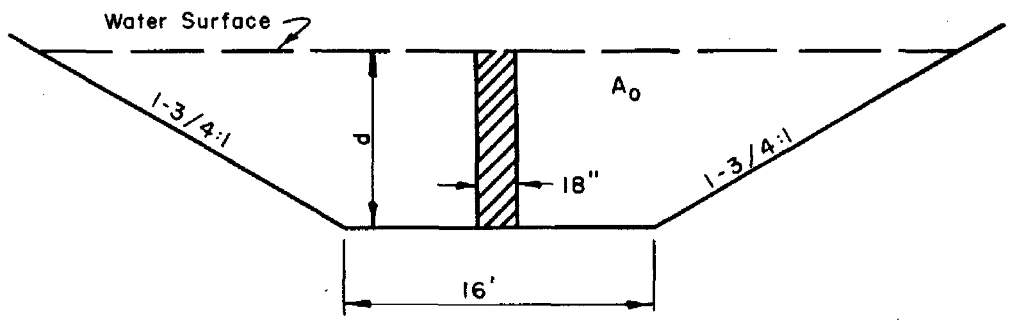


Fig. 5.2-2. Cross-section of channel showing center bridge pier.

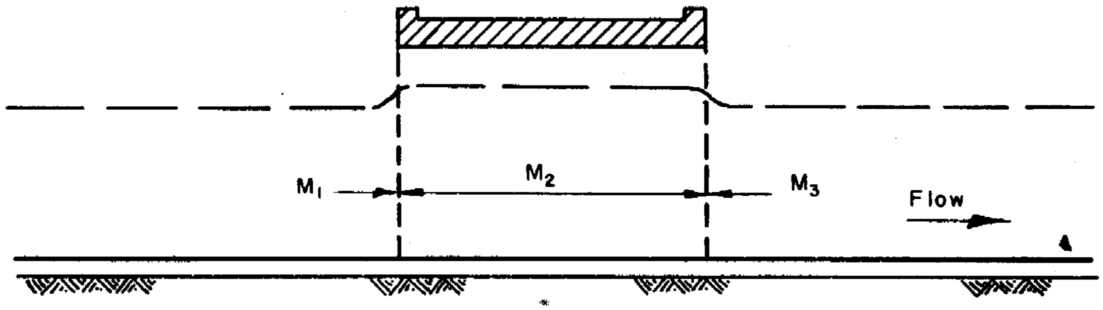


Fig. 5.2-3. Longitudinal section along centerline of channel indicates three locations: I - immediately upstream, II - Under bridge, III - Immediately downstream.

Compute M_1 - M_3 by solving for the distance $\bar{y}$ from the water surface to the center of gravity of the trapezoidal section where $\bar{y} = \frac{d(T + 2b)}{3(T + b)}$.

(T = top width and b = base width) and multiplying $\bar{y}$ by the area A, or ($M_1 = \bar{y}\,A_1$). The $\bar{y}$ distance for the obstruction, which is of rectangular area A_0 is $\frac{d}{2}$; this multiplied by A_0, will give M_0.

Quantities for the remaining columns can be easily computed by use of the formulas given on pages 5.2-2 and 5.2-5 and in the column headings in Table 5.2-1.

Momentum values given in columns I, II, and III, Table 5.2-1, are shown plotted on the graph, Fig. 5.2-4, giving curves I, II, and III. From a point on Curve I, which represents the upstream depth d_1, draw a vertical line through Curves II and III which will give the depth values for the sections under the bridge and immediately downstream. For the example given, refer to the curves on Fig. 5.2-4. For an upstream depth of 8.0 feet, the depth under the bridge is 9.3 feet and the depth immediately downstream is 8.7 feet. As the flow upstream is at subcritical depth, d_1 is less than d_c and "Class C" flow applies (see Fig. 5.2-1).

It is shown in Table 5.2-1 that the bridge section includes an 18-inch wide pier. As debris piles up on the center pier its net effect is to widen the center pier increasing A_0 and M_0. The effect of such debris accumulations can be estimated by computing flow conditions for the wider pier that would result when debris had accumulated. In critical cases the effect of debris lodging against piers can be minimized by constructing a 2:1 incline on the upstream edge of the piers. This causes debris to rise toward the surface and widen only a portion of the pier height. It should be noted, however, that the top eight (8) feet are normally considered to be affected by such debris so an inclined leading edge on piers in shallow streams would not be too effective.

The momentum method of computing the approximate change of water surface is not dependent upon coefficients "K" as are necessary in the formulas derived by Nagler, Weisbach, Rehbock and others (see USDA Technical Bulletin No. 429, "Pile Trestles as Channel Obstructions" and USDA Technical Bulletin No. 442, "Bridge Piers as Channel Obstructions").

A check computation for the raise in water surface due to the bridge obstruction was made using the Nagler formula. The coefficient K varies from .87 to .94 with the channel contraction approximately five percent. The difference in water surface elevation between the depth in the unobstructed channel and the depth caused by the obstruction was from 0.7 to 0.8 foot. Difference in depths indicated by the Momentum Method was greater, shown by the curves to be 1.3 feet, and is on the conservative side.

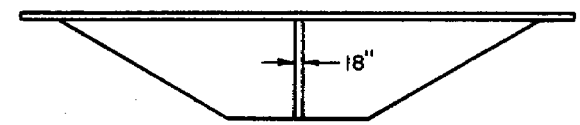
The water surface profile, above and below bridges, can be computed by the standard step method, as described in a report "Technical Memorandum - Water Surface Computation in Open Channels" by R. F. Wong, Los Angeles District Corps of Engineers, and also given in King's Handbook of Hydraulics.

Table 5.2-1 MOMENTUM LOSSES AT BRIDGES

Project Bull Creek Channel | Locality Bridges over channel

Q = 5,000 c.f.s. | Sheet 1 of 1 | By R.M.J. | Date February 1953

Section



d	$A_1 = A_3$	$M_1 = M_3$	A_0	M_0	$A_1 - A_0$	$M_1 - M_0$ (1)	$\frac{Q^2}{g(A_1 - A_0)}$ (2)	$\frac{Q^2}{g\ A_1^2}$	$\frac{Q^2 (A_1 - A_0)}{g\ A_1^2}$ (3)	$\frac{Q^2}{g\ A_3}$ (4)	I (1) + (3)	II (1) + (2)	III (1) + (4)
4	92	165	6.0	12.0	86	154	9028	92	7889	8440	8043	9182	8594
5	124	274	7.5	18.8	116	255	6664	50	5908	6261	6163	6919	6516
6	159	413	9.0	27.0	150	386	5176	31	4607	4883	4993	5562	5269
7	198	592	10.5	36.8	187	555	4141	20	3723	3921	4278	4696	4476
8	240	811	12.0	48.0	228	763	3405	13.5	3073	3235	3836	4168	3998
9	286	1075	13.5	60.8	272	1014	2849	9.5	2591	2715	3605	3863	3729
10	335	1384	15.0	75.0	320	1309	2426	6.9	2214	2318	3523	3735	3627
11	388	1746	16.5	90.8	371	1655	2090	5.2	1924	2001	3556	3745	3656
12	444	2158	18.0	108	426	2050	1825	3.9	1680	1749	3730	3875	3799
13	504	2636	19.5	126.8	484	2509	1602	3.1	1482	1540	3991	4111	4049

M_1 = Static moment in unobstructed channel

M_0 = Static moment of obstruction

$M_1 - M_0$ = Static moment in obstructed channel

$$M_1 - M_0 + \frac{Q^2}{g\ A_3} = M_1 - M_0 + \frac{Q^2}{g\ (A_1 - A_0)} = M_1 - M_0 + \frac{Q^2\ (A_1 - A_0)}{g\ A_1^2}$$

$\frac{Q^2}{g\ (A_1\ \ A_2}$ = Kinetic momentum in obstructed channel

$\frac{Q^2}{g\ A_1}$ = Kinetic momentum in unobstructed channel

$\frac{A_0\ Q^2}{g\ A_1^2}$ = Kinetic momentum lost

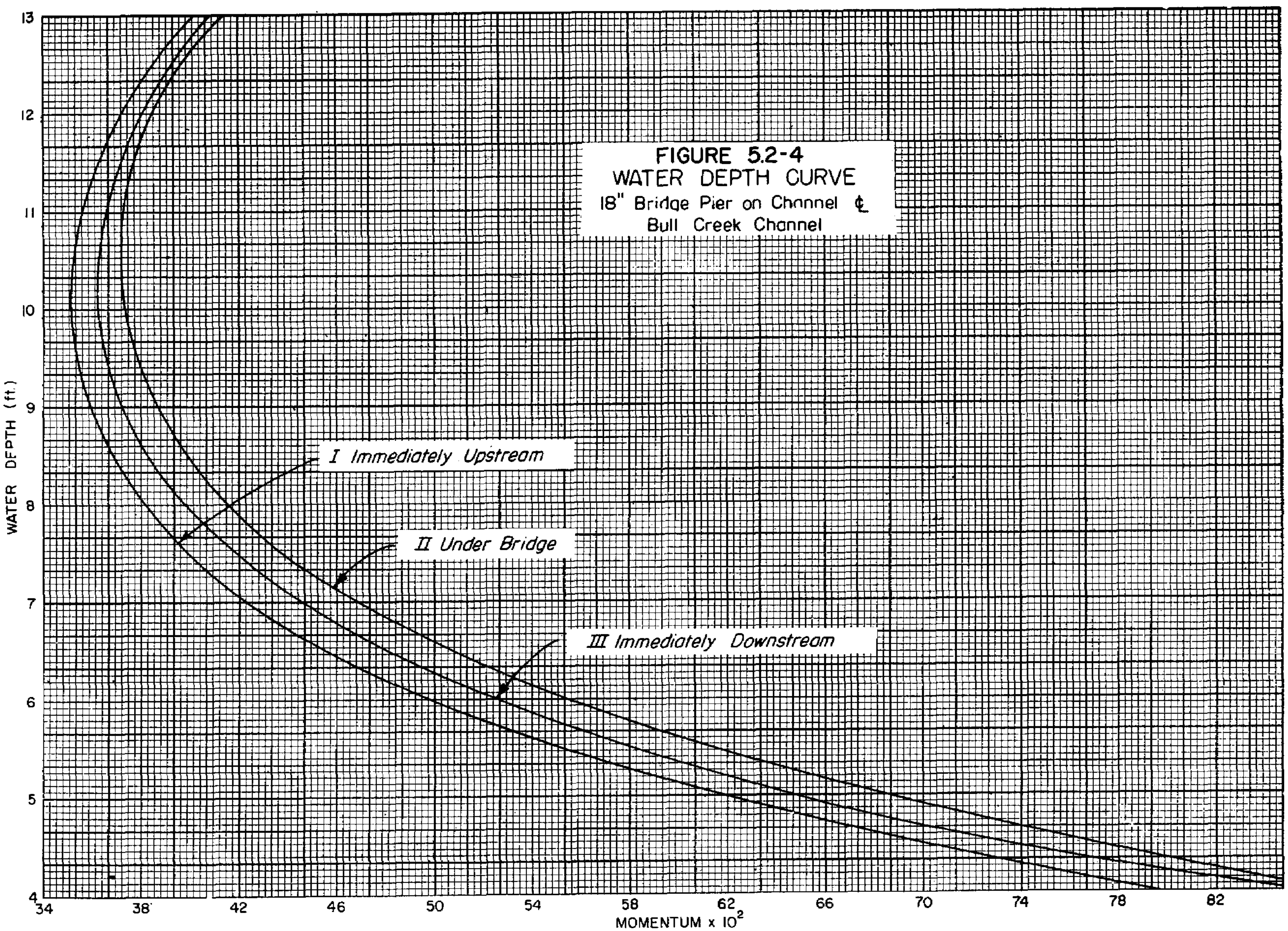


FIGURE 5.2-4
WATER DEPTH CURVE
18" Bridge Pier on Channel ℄
Bull Creek Channel
I Immediately Upstream
II Under Bridge
III Immediately Downstream
WATER DEPTH (ft.)
MOMENTUM x 10²
13
12
11
10
9
8
7
6
5
4
34
38
42
46
50
54
58
62
66
70
74
78
82